CAWOOD

Detailed Study of
Impact in Steel Bridges

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DETAILED STUDY OF IMPACT IN STEEL BRIDGES

BY

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

HERVEY RICHEY CAWOOD

ENTITLED DETAILED STUDY OF IMPACT IN STEEL BRIDGES

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Bachelor of Science in Civil Engineering

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I. INTRODUCTION.

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For many years engineers have been seeking a suitable formula for computing the stress in bridge members due to the passage of trains over the bridge.

If every element which has some connection with the effect produced on a bridge by a fast moving train had to be considered the problem of impact would be insoluble even in the simplest case. By having such elements as, for instance a defective track or inequalities of the rail end at rail splices or flat wheels whose influence is naturally outside the province of a calculation, the mathematical difficulties presented by the problem are almost unsurmountable. In fact have only been overcome for the case of a single load moving over a beam.

A train in passing over a bridge causes the latter to deflect, whereby the pressure or centrifugal force exerted by the train against the bridge is influenced by the deflection and the velocity of the moving masses. In consèquence of the great velocity with which a train enters a bridge, of the variable laods produced by the counterweights of the locomotive, of defective rail splices and flat wheels, the bridge is subjected to vibrations, which cause increased strain.

As it is impossible to determine these strains mathematically only a theoretical discussion can be given,

proving that in case of single loads, there are stresses due to impact. Experiments alone can determine the actual impact in bridge members due to moving trains. As there is such great possibility of instrumentals errors due to the delicacy of the instruments and the sudden vibrations of the bridge, a large number of tests must necessarily have been made.

A committee appointed by the American Railway Engineering and Maintenance of Way Association have made about 15 000 such tests. From the results of these tests the conclusion of this thesis has been drawn.

II. THEORY OF IMPACT.

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Sudden Loads and Stresses.

A load at rest on a bar or beam, or one which increases from zero up to its final value P in such a way that the deformation at different instants are proportional to the loads acting at those instants until the elastic limit of the material is exceeded, is called a static load. Loads applied in any other manner are sometimes called " dynamic" loads, and the term " impact" implies either suddenness of action or that the load is in motion before it is applied to the bar or beam. The terms " dynamic" stress and " dynamic" deformations are used to distinguish the effect of impact from those due to static loads.

If a static tensil load is applied to a bar by increments, so that it increases from zero up to P in such a way that the elongation is proportional to the load until the elastic limit of the material is reached, the work done upon the bar will be equal to 1/2 P times the elongation e, or $K = 1/2\ P\ e$. At the same time the stress in the bar increases from zero up to P and the internal energy stored in the bar is then equal to $1/2\ P\ e$. The triangle in Figure 1 represents both the external work and the internal energy.

When the load is applied to the bar in such a manner that its intensity is the same from the beginning to the end

of the elongation, it is called a "sudden load". For instance, let a load be hung by a cord and just touch a scale pan at the foot of a vertical bar; then if the cord is quickly cut, the load acts upon the bar with uniform intensity thruoghout the entire elongation. In this case the maximum elongation is greater than for a static load, but the bar at once springs back, carrying the load with it, and a series of oscillations insues, until finally the bar comes to rest with an elongation due to the static load. Here the stress in the bar increases from zero up to Q, the stress Q being equal to the static load which would produce the maximum elongation. Fig. 2 represents this case, where the rectangle shows the work done by the instaneous load P, and the triangle shows the internal energy stored in the bar at the instant of greatest elongation. The unit stress due to Q must be less than the elastic limit in order that the following discussions may be valid.

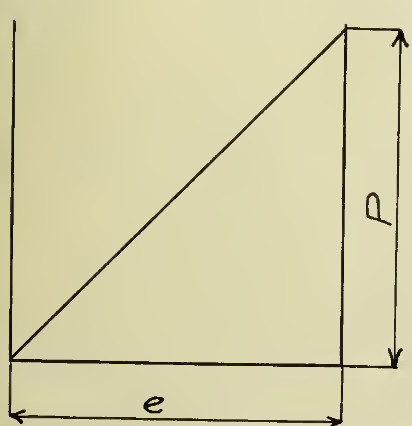

Fig. 1

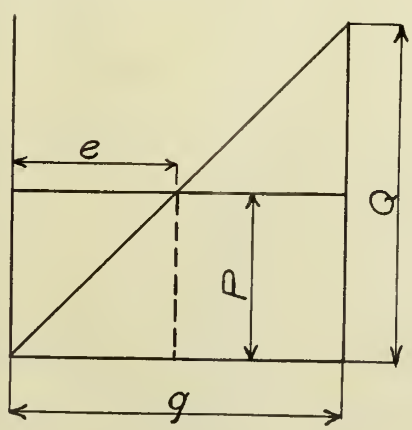

Fig. 2

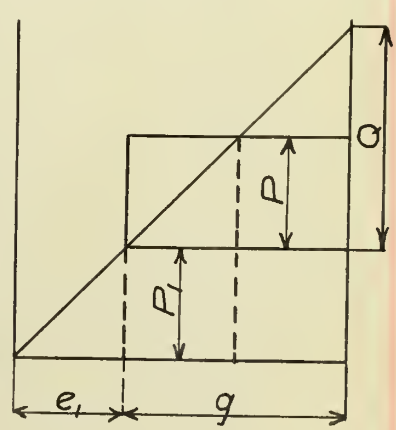

Fig. 3

Let q be tha maximum elongation due to the sudden load P, the work performed in the bar is P q, the internal energy stored in the bar at the instant of greatest elongation is 1/2 Q q since the stress increases from 0 up to Q . Hence 1/2 Q q = Pq or Q = 2 P . Let e be the elongation due to the static load P ; then Q/e = Q/P , q =2e . Accordingly the following important law is established for a bar under elastic changes of length;

A sudden load produces double the
stress and double the deformation
that is caused by a static load.

In the above discussion P and Q are the total stresses coming upon a bar with a sectional area of <u>a</u>. Let S and T be the corresponding unit stresses, so that P = S <u>a</u> and Q = T <u>a</u> . Then the equation Q = 2 P becomes T = 2 S, that is, the unit stress due to asudden load is double that due to the same static load.

Similar conclusions were drawn from experiments performed by the writer upon wooden beams resting upon supports 70 inches apart. The beams were approximately 2"x 1 1/2" yellow pine quality A - 1, The loads were applied at the center by increments and upon the addition of each increment the static deflection was first measured, then the load was raised and applied suddenly by falling through a distance of 2 inches.

Let Q by the static load which produced the deflect-

ion q, and P the same load applied suddenly which produced the deflection f . The experiment showed that Q q = 2 P f , or q = 2 f approximately that is, the dynamic deflection is double the static deflection. From this the same unit- stress relation-ship is obtained namely, that the unit-stress due to sudden loads is double that due to the same static load.

Lastly, consider a bar upon which rests a load P_1 causing the elongation e_1. Let a sudden load P now be brought upon it causing the additional elogation q and the additional stress Q. Fig. 3 represents this case and it shows that the elongation is $e_1 + 2e$ and that the final stress is $P_1 + 2P$; thus the instantaneous load produces its effects independently of the other. As soon as the elongation $e_1 + 2e$ occurs the bar springs back, and a series of oscillations follows; finally the bar comes to rest under the elongation $e_1 + e$ and the stress $P_1 + P$.

In the above investigations it has been supposed that all the work P q performed by the sudden load p is expended in storing energy in the bar or beam. This is not strictly the case, as was shown in the experiment with the beam, the slight loss of work being attributed to the internal molecular friction. The law deduced is, however, very close for a light beam, but Q is really a little less than 2 P and Q a a little less than 2 e when the beam is heavy compared with the load.

When a falling weight strikes a beam it causes a greater deflection than a load suddenly applied. Let a weight P fall from a height h above a light and produce the dynamic deflection q , the work performed is then P(h + q). Let T be the maximum flexural unit stress produced by the impact and S be that due a static load P which causes the deflection f . Then the deflections are proportional to the unit stresses, if the elastic limit is not exceeded, or q/f = T/S . Also let Q be a static load which will produce the deflection q ; then the deflections are also proportional to the loads, or q/f = Q/P; accordingly Q/ P = T/S . The external work of the load Q is 1/2 Q q and this is equal to the internal energy stored in the beam when the deflection Q is attained, if all the work is expended in stressing the beam. Hence 1/2 Q q = P(h + q) , which by above ratio reduces to 1/2 T q = S (h + q) . Combining this with q/f = T/S , there are found,

$$T = S + S\sqrt{(1 + 2h/f)} \qquad \text{and}$$

$$2q = f + f\sqrt{(1 + 2h/f)}$$

When a weight P is moving with the velocity V, it can perform in coming to rest the work $P V^2/2g$, where g is the acceleration of gravity. When the weight moves horizontally and strikes normally against the side of a beam which has its ends arranged so as to prevent lateral motion, a lateral dynamic deflection results.

Let h be the height corresponding to $v^2/2g$, then the external work P h is equal to 1/2 Q q , from which the following relations are obtained,

$$T = S\sqrt{(1/2h/f)} \quad , \text{ and}$$

$$q = f\sqrt{(2h/f)} = \sqrt{(2hf)}$$

for the unit stress and lateral deflections at the instant P comes to rest.

The above discussions have dealt with the effects of the application of comparatively light loads upon small bars and beams. The relations of the dynamic stress and dynamic deflection of the sudden load to the stress and deflection of static loads have been determined. It remains now to investigate the effects of great loads suddenly applied to beams.

III. SHOCKS ON RAILWAY BRIDGES.

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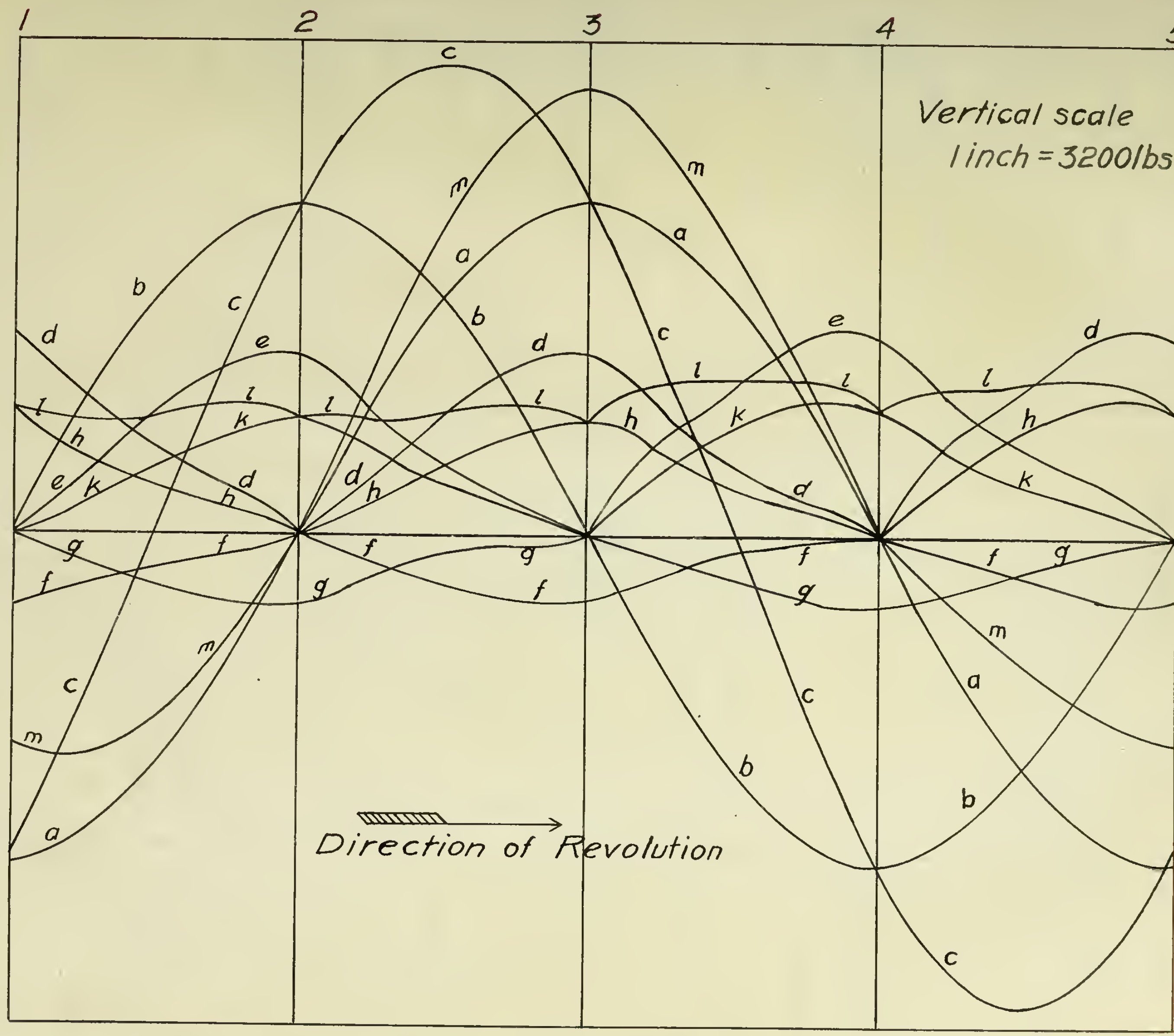
In order that a locomotive may run at high rates of speed without fore and aft irregularities of motion it is necessary that the counterweights be added to the driving-wheels, not only for crank-pin hold, crank-pin, and weight carried thereon, but also for the weight of those parts which move only in a horizontal direction viz, piston, piston-rod, cross-head, and that part of main rod carried by the cross-head. This additional counterweight is usually divided equally between all the driving wheels and, in the case of a Pennsylvania Railroad standard locomotive, class "B", it requires and addition of counterweight to each wheel equivalent to 300 pounds at a distance of 12 inches from the wheel center to properly control the motion. This brings correspondingly too much counterweight vertically, but it does not result in objectionable disturbances because the forces are resisted by the road-bed in one direction and by the weight of the machine in the opposite direction.

In locomotives for low speed only a part of this additional counterweight can be left out without apparent evil results, but in express locomotives it is necessary to use nearly all that theory requires. From this cause a series of blows results, which may be examined, a little more in detail by the aid of the accompanying plate.

PLATE I



Note: No's. 1 & 5 Counter-weight in Highest Position. No. 3 Counter-weight in Lowest Position

The length of this diagram, from left to right, represents one revolution. The vertical scale is one inch equal 32,000 pounds, and the horizontal line through the center is a line of normal pressure of one wheel upon the rail, i.e. it is a sufficient distance above an imaginary datum line below, and entirely off the diagram, to represent, on the same scale the quiescent weight of the wheel on the rail. The speed is assumed at 50 miles per hour, and one revolution is considered commencing when the engine on the right side is in its first quarter, that is when the counterweight on that same side is in its highest position. The curve "a" shows the boundary of the vertical components of the centrifugal force of 300 pounds additional counterweight at a radius of 12 inches for one wheel on the right side of the locomotive, these components being laid down, on a scale of one inch = 3200 pounds, from the line of normal pressure, below or above, according as they are directed upward or downward, and so diminish or increase the load of the wheel upon the rail. It will be born in mind that this additional 300 pounds of counterweight, if located, as supposed, with its center of gravity 12 inches from the wheel center (the same as crank radius), will move horizontally at all times with the same rate of speed and in opposite direction to the parts it is employed to balance; if it is of any other weight it must be placed at some other distance such that its energy of motion horizontally and vertically will at all times be the same.

Therefore the curve given still shows the disturbance of vertical pressures.

The curve "b" shows similarly for one wheel on the left side. This curve is identical with that already explained, only one quarter revolution in advance, as the engine on the left side leads the right by that amount. The line "c" shows the resultant of these two curves, and is for two wheels on one axle, but the curves"a" and "b" must be considered individually as to their effects on ties, rails, and roadway structures.

It will be seen that from this cause their results an increase of 6,260 pounds above the normal and a decrease of same amount below the normal in the weight of each driving wheel upon the rail every revolution at the assumed speed of 50 miles per hour, and this cycle is repeated four and one-half times every second, so that it is a series of quick blows of magnitude 2 x 6,260 = 12,520 pounds; it is needless to add that for other speeds, this will increase or decrease with the squares of the speed, and at 60 miles per hour, it will be 44% greater.

Another source of vertical disturbance, and one quite different from that just detailed, is that disturbance resulting from the application to the main pair of wheel only, or that pair to which the engines are directly connected.

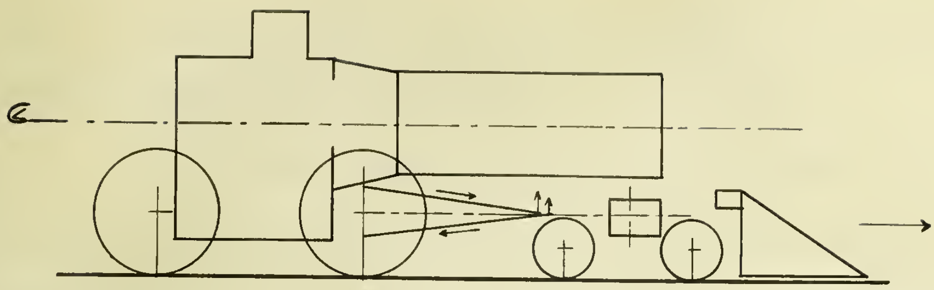


Fig 4

Refering to Fig. 4 which shows of the out lines and center lines of Pennsylvania Railroad, class "B", locomotive, it will be observed that when it is running forward the main rod is pulling obliquely downward on the crank-pin when the latter is above the centre line of cylinder, and that it is pushing obliquely downward on the same pin when the latter is below this line. Therefore except on the two centres, the power of the engine is directed more or less downward on the pins at all times. So far as this goes it increases the weight of the wheel upon the rail by so much as the vertical component of this bearing force with its varying obliquity is extended through the main rod.

On Plate I again, the curve "d" above the horizontal line of normal pressure, is the boundary of the vertical components of the force exerted by connecting-rod on the crank-pin during one revolution. This curve lies wholly above the line of normal pressure, except two points which are

in this line, because, as already stated the vertical component, when there is any, is always directed downward in running forward, and therefore increases the pressure on the rail.

In plotting these curves the conditions prevailing in this class of locomotives have been observed, with the assumption that the cut-off occurs at one-half the stroke, and that the cylinder pressure, up to time of cut-off, is 110 pounds per square inch.

The corresponding line "e" shows the same for the opposite main wheel, being one quarter revolution in advance, as before. As this vertical force is consequent upon exertion through mechanism of a horizontal force in the cylinder, there must be at all times an equal reaction vertically upward; this is by the cross-head thrust against it upper guide, and so far as this force goes it tends to reduce the aggregate weight of the machine on the rails by just so much as we find it increased at the main driving wheel at all times. The upward force, however, from the varying location of its points of application, is variously distributed at different times as to its relieving effects from driving wheels and leading truck-wheel. Disregarding the relief from the rear driver, and for the right side, supposing it all distributed, between the main wheel and the truck it will be found, in plotting, that the curve "f" below the line of normal pressure is the boundary of relieving forces at the main wheel for one wheel, the remainder going to relieve the weight on

the truck. The corresponding line "g" shows the same thing for the left side, and is one quarter revolution in advance. Taking the resultants of the two opposed forces for each side separately we have for the right side, the line "h" and for the left side the line "k" while the line "l" is the resultant for both sides. On the assumption above, the main driving wheels are loaded by this amount at the expense of the truck, through the action of the mechanism. They are really loaded more than the diagram shows, because the relieving action from the rear drivers has been disregarded as some what indefinite in magnitude.

The upward reaction on the guides causes the machine to roll, as is often seen when it is laboring hard at slow speed. If the locomotive runs backward the conditions are reversed and main wheels are relieved, the weight being transferred to the truck, so that the weight available for tractive power is less in running backwards than it is in running forward.

This concentration of weight on the main drivers varies only with variation of pressures in the cylinder, and is therefore, independent of speed except as these pressures are varied in consequence. At high speeds this also assumes the character of quick blows on each side and of course at fifty miles per hour the cycle is repeated 4 1/4 times per second as before.

Considering all these disturbing forces on one main wheel only, for the right side, the curve "a" shows the

disturbance from counterweight and the curve "h" shows the resultant of disturbing forces from the connecting rod, while the curve "m" shows the resultant of the two on a scale 1 inch = 3,200 pounds.

Good practice keeps the count rweight as low as possible, but on express locomotives the disturbances here shown frequently occur at each main wheel and rear wheel respectively at the same time . For each main wheel this is an increase of weight upon the rail of 8350 pounds above the statical weight frequently figured on in designing structures, and this increase is followed by a decrease below the normal of 4,250 pounds, making a variation of 12,600 pounds which, at 50 miles per hour is repeated 4 1/2 times per second.

For each rear wheel it is an increase of weight upon the rail of 6260 pounds, followed by a decrease of the same amount, making a variation of 12,520 pounds 4 1/4 times per second at the same speed, and these variations on one rail are synchronous, i.e. are going through the same phase at the same time; there is of course a slight release of weight from the truck at the same time, but this never exceeds 2,600 pounds.

This is therefore, a sreies of blows each one directed upon some point and it requires but the proper combination of circumstances as to the situation of this point (the location of the other point on same rail being struct at the same time), and the proximity of other heavily loaded wheels to produce the maximum strains upon some member or members of a resisting

structure already in a state of oscillation from regular and successive blows received. This will account for the necessity of a suitable formula that may be used in in computing the strains which are produced upon different members of a bridge by the train when running above a certain speed.

IV. COMPARISON OF FORMULAE.

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Not until recent years has it been thought necessary to take into account the stress produced in bridge members by vibrations and deflections set up by moving trains. Since 1895 extensive experiments have been carried on by the American Railway Engineering and Maintenance of Way Association which lead that body to adopt the formula (known as Schneider's formula)

$I = \frac{300}{300 + L}$ I = percent impact. L = length of bridge loaded when maximum stress is produced in member under consideration.

All tests that have been made have led to the conclusion that the additional stress due to a train moving with a velocity of 20 miles or less in considerable,but with a greater velocity the increased stress due to impact varies from ten to ninety-five percent of the live load stress, depending upon the member and the length and kind of bridge.

The results of a few of the many experiments made by this association between 1900 and 1905 are given in Table 1. The plate girder experimented on were deck and the instruments were attached to the lower flange. The trusses were all thru pin connected with stiff end verticals and end bottom chords. The counters were adjustable,the floor beams were riveted to the posts and the stringers to webs of floor beams. The bridges were all modern structures and in good condition.

The results here given are the minimum and maximum of all tests made on individual bridges and are for high speeds.

Table I.

IMPACT PERCENTAGES.

Class of Structure.	Span or Length of Loading.	Measured Impact Percent.	Impact Schneider's Formula $I = \frac{300}{L + 300}$ Percent.
Plate Girder.	51 ft. 6 in.	15 to 85	91.
	60 ft. 3 in.	36 to 71	83.
	75 ft. 6 in.	18 to 61	80.
	Bottom Chords.		
	100 feet.	16 to 57	75.
	153 "	11 to 32	66.
	207 "	18 to 50	59.
Trusses.	Main and Counter Ties.		
	40 feet.	16 to 58	88.
	60 "	20 to 60	83.
	76 " 6 in.	9 to 15	80.
	114 " 9 "	5 to 49	73.
	161 "	4 to 15	65.
	Hangers.		
	38 feet.	8 to 56	88.
	46 "	17 to 32	87.

It is seen from the above table that the impacts as computed from the formulae are greatly in excess to those actually determined.

Two formulae that have been extensively used and which may be compared with that of Schneider's are those of Pritchard, and of Cain, the latter being known as the Pennsylvania Railroad formula. These are:

$$\text{Pritchard, } I = \frac{ls}{ls + ds}.$$

$$\text{Cain, } I = \frac{max.}{max. + min.}$$

Cain's formulae in order that it may be compared with the other two, is written:

$$I = \frac{ls + ds}{ls + 2ds}$$

In the above:

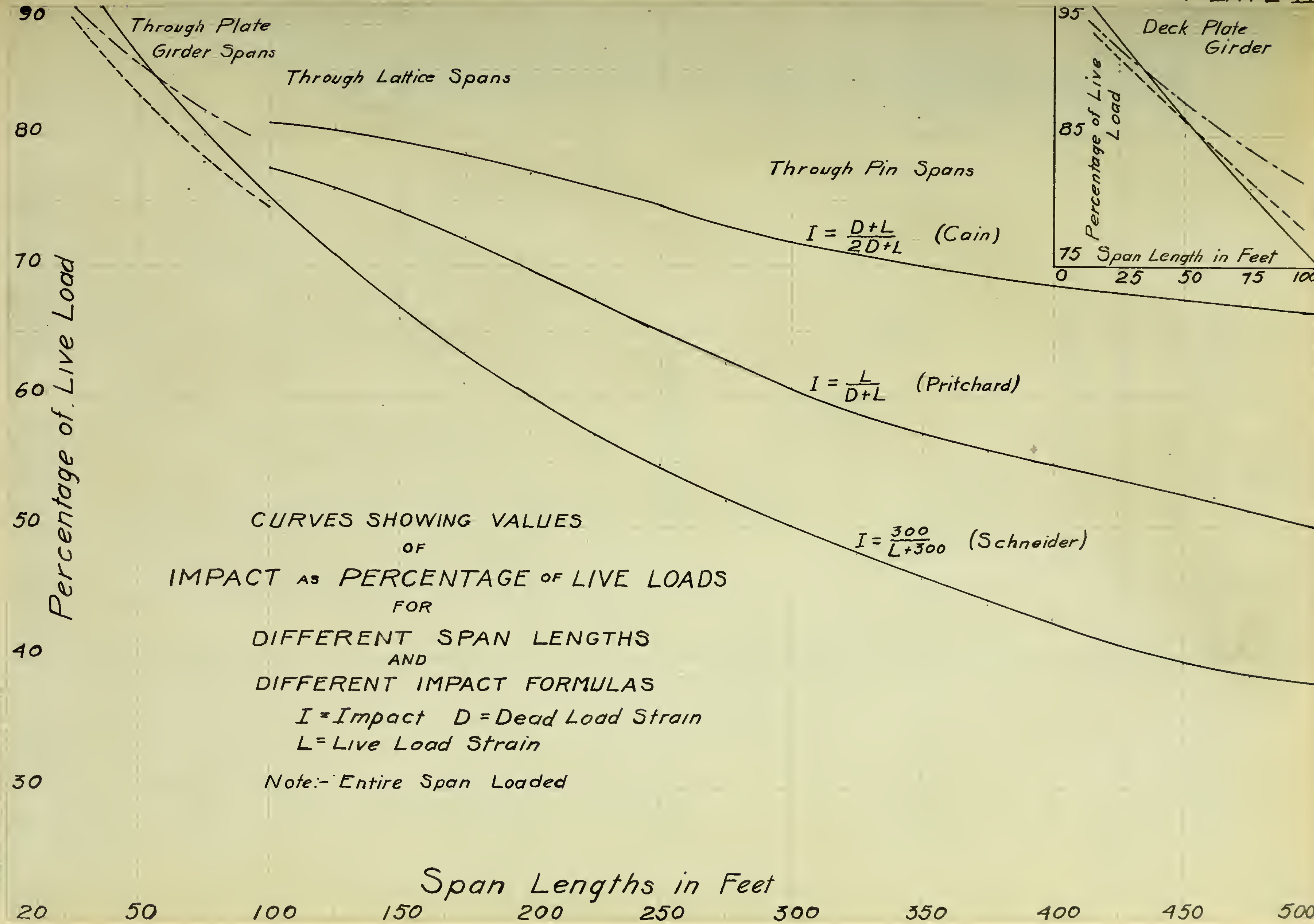
I = percent impact.

ls = max. live load stress.

ds = dead load stress.

These three formulae may be best compared by plotting the percentages of impact for various spans as computed by each one of the three. From the curves Plate II it is seen that for all truss bridges the percentage of impact as determined by Prichard's and Cain's formulae is higher than that computed by Schneider's formula. For girder bridges the curves show very little difference in the impact as determined by the three formulae. As Schneider's formula is the one most extensively

PLATE II


Through Plate
Girder Spans
Through Lattice Spans
Through Pin Spans
$I = \frac{D+L}{2D+L}$ (Cain)
$I = \frac{L}{D+L}$ (Pritchard)
$I = \frac{300}{L+300}$ (Schneider)
Deck Plate Girder
Percentage of Live Load
Span Length in Feet
95
85
75
0
25
50
75
100
CURVES SHOWING VALUES
OF
IMPACT AS PERCENTAGE OF LIVE LOADS
FOR
DIFFERENT SPAN LENGTHS
AND
DIFFERENT IMPACT FORMULAS
I = Impact D = Dead Load Strain
L = Live Load Strain
Note:- Entire Span Loaded
Percentage of Live Load
90
80
70
60
50
40
30
Span Lengths in Feet
20
50
100
150
200
250
300
350
400
450
500

used, being specified in a number of prominent specifications, and as it seems to give values approaching more nearly the true values of impact percentages than either of the others, it will be used in the following comparisons.

V. RESULTS OF IMPACT TESTS.

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During the summer of 1907 a committee, authorized by the American Railway Engineering and Maintenance of Way Association, in charge of F. E. Turneaure made a large number of tests upon bridges for the purpose of determining the actual impact.

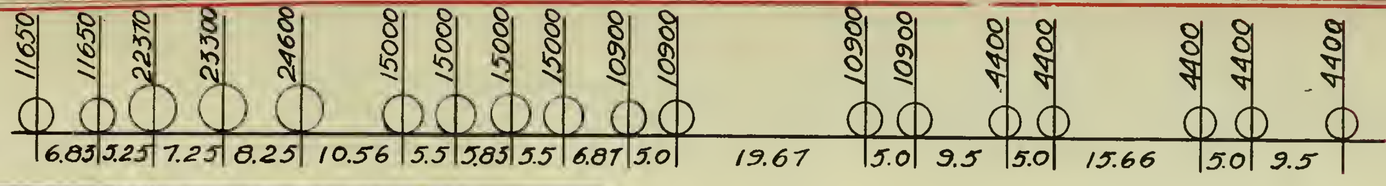
Tables II and III show the results of their tests upon a 132 foot, pin connected bridge. The loading was as shown above each table.

Owing to the limited amount of time given the committee it was impossible for the tables given above to be checked and the instrument diagrams to be studied as they should be before a final report could be made. However, it is very evident that the percentage impact as determined by Schneider's formula is for in excess of the actual impact percentage as determined from the instrument diagrams.

The relation of Prichard's and Cain's formulae to Schneider's and the evident excessive values of impact as determined by the latter are sufficient proof of the uneconomical design obtained when these formulae are used in obtaining the stress due to impact.

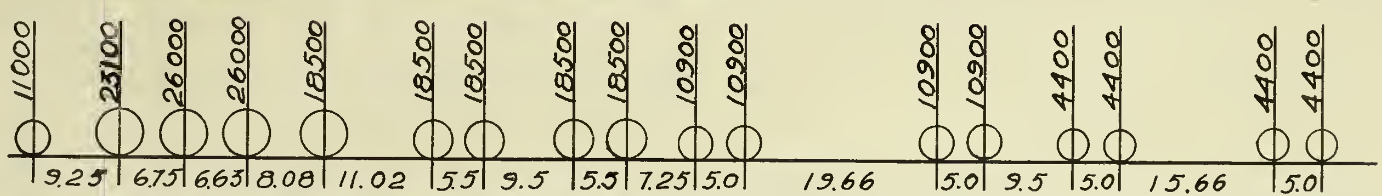
The committee of 1907 continued their work through the summer of 1908. From the tests made during the two years, numbering about 15,000 some very satisfactory results have been

TABLE II



Inst		Deflection				1				2				3				4				5				6				7	8
Where Located		Bottom Chord Center P. Point				Main Diagonal West				Stringer 3rd P East				Lower Chord 3rd P. Bottom Inside				Lower Chord 3rd P. Top Inside				Lower Chord 3rd P. Top Outside				Hip Vet. Inside Bar West Edge					
REC.	SPEED MPH	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAK	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP		
236	48	124	146	126	18	60	71	61	18	45	50		11	39	45	40	15	30	38	33	27	27	35	29	30	58	70	63	21		
237	48	124	135	132	9	60	69	64	15	45	55		22	39	45		15	30	34	29	13	27	32	28	19	58	80	63	38		
240	44	124	147	127	19	60	70	60	17	45	53		18	39	50		28	30	36	31	20	28	33	28	22	58	70		21		
																		Lower Chord 3rd Top Outside				Lower Chord 1st Bottom Outside									
248	28	124	130	125	5	60	64	60	7	45	50		11	37	40		8	30	30		0	8	14		75	58	62		7		
251	36	124	140	122	13	60	70		17	45	49		9	37	41		11	28	35		25	8	11		38	58	69		19		
253	37	124	145	124	15	60	75	67	25	45	50		11	35	44		26	28	33		18	8	10		25	58	69		22		
255	57	122	138	121	13	60	69	62	15	45	51		13	35	42		20	28	38		36	8	10	7.5	25	58	80		38		
259	40	120	143	121	19	60	71	57	18	45	52		16	35	41		17	26	34		31	8	10		25	58	63		9		
260	42	120	141	118	18	60	69	59	15	45	51		13	35	43		23	26	31		19	8	8		0	58	65		12		

TABLE III



Where Located		Bottom Chord Center P. Point				Lower Chord 3rd P. Top Inside				Stringer 2nd P South Flange				Stringer 3rd P South Flange				Hip Vert. Inside Bar West				Lower Chord 3rd P. Outside				Main Diag. West				Stringer 2 P. North Flange				Stringer 3rd P North Flange			
REC.	SPEED MPH	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP	STATIC	MAX	MEAN	%IMP
267	20	148	150		1	21	25		19	57	65		14	51	52		2	67	71		6	41	47		15	78	78		0	54	59		9	50	51		2
271	39	144	188	148	31	25	23		32	57	55			51	62	41	22	67	87		30	41	66	53	61	76	103	78	36	50	68		36	50	61		22
280	47	142	158	138	11	25	28		12	57	109	74	91	51	73	48	43	67	88		31	41	55		34	76	86		13	50	50		0	50	62		24
283	35	140	170	145	21	25	29		16	57	72	52	26	51	66	46	29	67	81		21	41	52	44	27	74	95	77	28	50	70		40	50	60		20
285	34	140	174	141	24	25	30		20	57	85	60	49	51	78	58	53	67	88		31	41	59	49	44	74	102	82	38	50	78		56	50	70		40
286	40	138	170	140	23	28	35		25	57	88	58	54	51	70	50	37	67	82		22	41	59	49	44	74	93	78	26	50	78		56	50	76		52
288	40	138	170	135	23	28	46		65	57	80	57	40	51	65	50	27	67	88		25	41	58	45	41	74	100	72	35	50	72		44	50	65		30
289	46	138	150	130	9	28	46		65					51	62	42	22	67	96		43	41	52	43	27	73	82	72	11	50	45			50	69		38
299	46	134	152	127	13	30	39	34	30	57	97	72	70	51	60	40	18	67	91		36	41	45	35	10	73	82		11	50	50			50	71		42
300	40	134	165	130	23	30	40	35	33	57	70		23	51	58	48	14	67	86		28	41	59	46	44	73	93	73	27	50	60		20	50	60		20
301	48	134	146	126	9	30	46	40	55	57	90		58	51	60		18	67	88		31	41				73	80		10	50	62		4	50	50		0

obtained. The diagram Plate III page 22a. has been drawn up from a few of the results of the 1908-tests. The bridges of 100 foot span and less represented upon the diagram were plate girder bridges. All the others were of the pin-connected truss type. The plotted values of impact show the effect of high speed trains upon short span bridges. The impact is found to vary from 7 percent for a 230-foot pin-connected bridge a speed of 25 miles per hour to 130 percent for a 48-foot thru plate-girder and a speed of over 60 miles per hour. It is quite probable that there were instrumental errors in the tests for the short span as such high percentages are very seldom obtained. In the greater number of these tests the percentage was found to be between 30 and 90 percent. These values correspond very closely with those given in Table I. For the truss bridges the percentage is found to increase from 7 to 48 according to the length of span and velocity of the trains.

With the plotted values for both plate girder and truss bridges it is possible to draw curves from the equations of which a fairly accurate theoretical percentage may be obtained. The curve $I = 10 + 40/l$ in which l is the span length when plotted is found to give percentages greater in most instances, than the actual percentage but for a 48-foot girder with high speed trains the actual percentage in one instance exceeds that from the curve by 30 percent in another instance there is a difference of 20 percent, that of the curve being the lower.

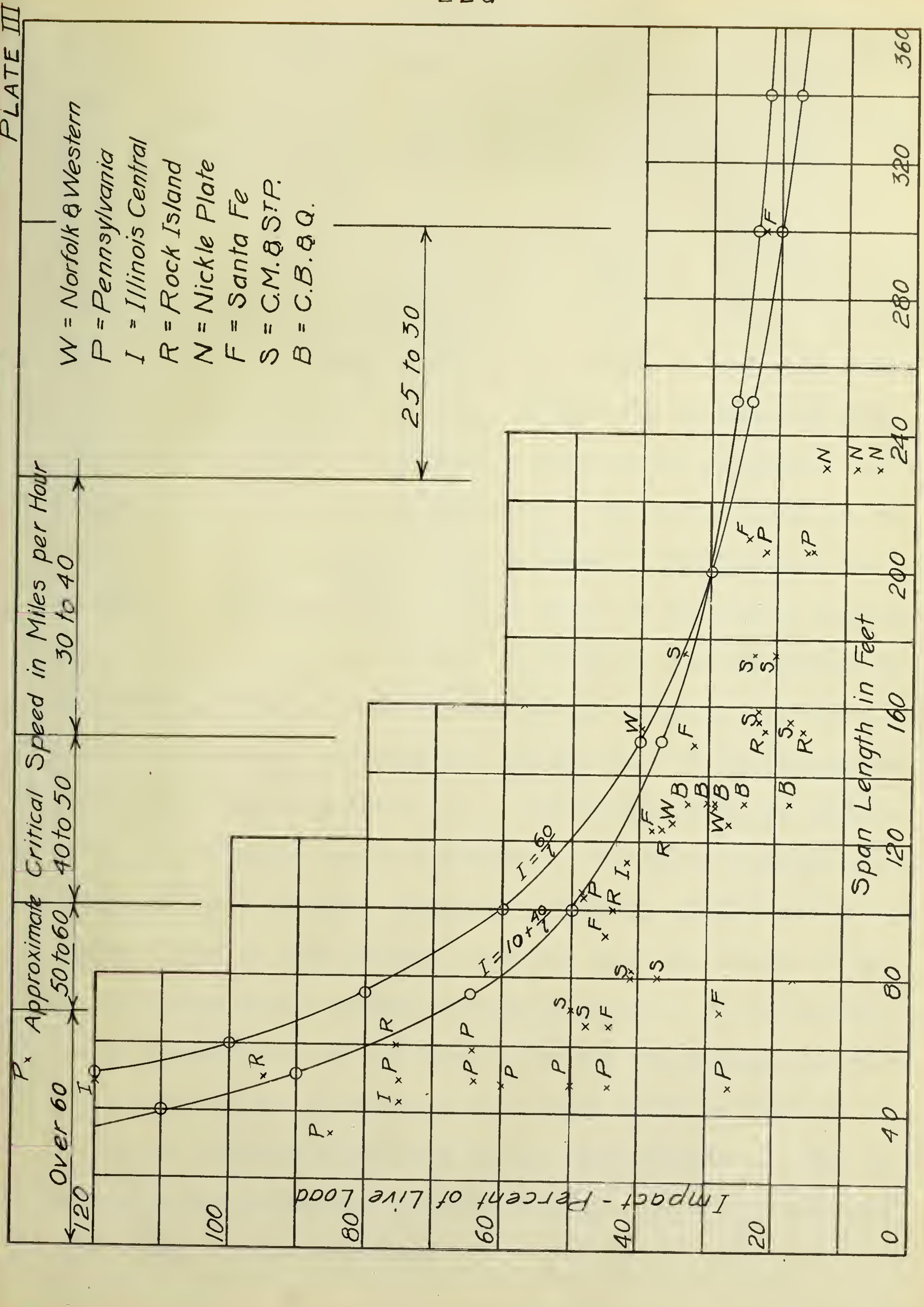


PLATE III
W = Norfolk & Western
P = Pennsylvania
I = Illinois Central
R = Rock Island
N = Nickle Plate
F = Santa Fe
S = C.M. & St.P.
B = C.B. & Q.
Approximate Critical Speed in Miles per Hour
Over 60
50 to 60
40 to 50
30 to 40
25 to 30
$I = \frac{60}{l}$
$I = 10 + \frac{40}{l}$
Impact - Percent of Live Load
Span Length in Feet
0
40
80
120
160
200
240
280
320
360
20
40
60
80
100
120

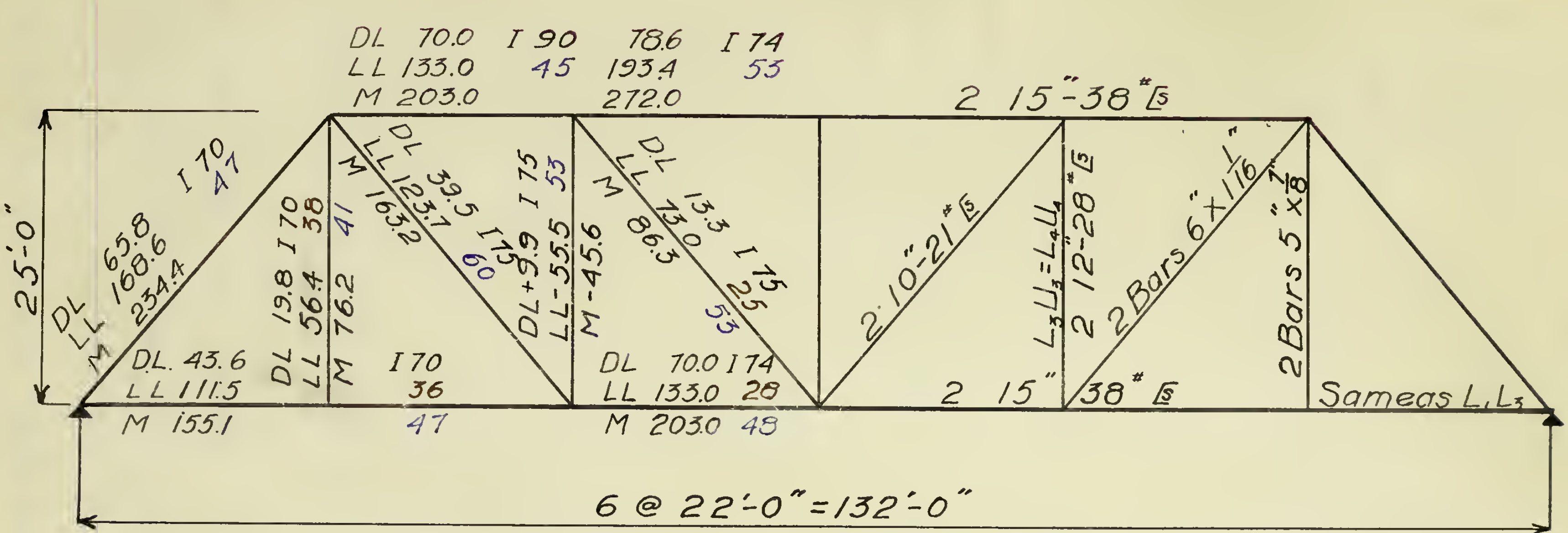
D

While it is probable that were errors which caused such high impact percentages, yet, until further investigations have been made and these errors proved it would be unsafe to use the formula $I = 10 + 40/l$.

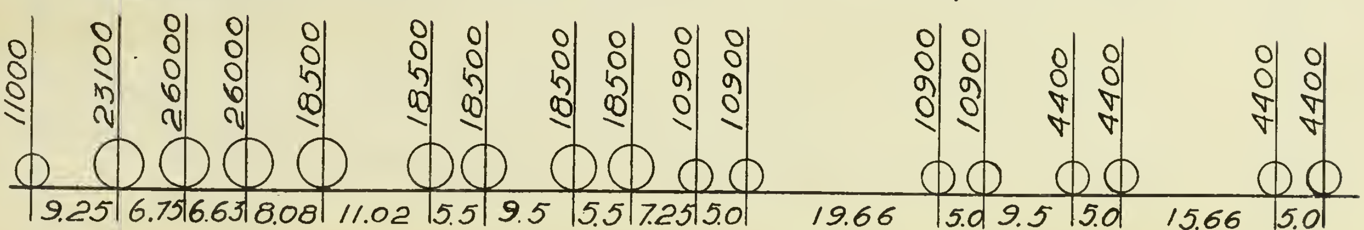
A curve which more nearly fulfills the actual conditions for impact is that plotted from the equation $I = 60/l$; where I is the percentage impact and l the span length in feet. The impact as determined by this equation may be compared with that from Schneider's formulae by computing the percentage for a particular bridge by the two methods. The bridge taken was a 132 foot span and loaded by a train as shown, the position for maximum loading being determined by means of the engine diagram. The percentage of impact written in black is that determined by Schneider's formula , that in blue by the formulae $I = 60/l$. The few percentages in brown are the maximum for the respective members as taken from Table III . The variation in the results by the two formula are very evident. The curve formulae gives a constant 45.5 percent, Schneider's formula gives results varying from 69.6 to 75.5 percent depending upon the length of the bridge loaded when a maximum stress is produced in the member. But by a comparison of the results for the few members for which the actual maximum percentages were taken there is found to be a sufficient percentage obtained by the curve formulae. For no member is the percentage obtained from the curve formulae less than actual tests have shown.

PLATE IV

132 Foot Pin-Connected Truss Bridge
Showing
PERCENTAGES of IMPACT



Loading - D.L 1800# Per Lin. Ft. L.L. Eng. No. 1919 C.B.& Q. R.R.



The curve diagram Plate 111 also shows results higher than the actual with a maximum velocity of 50 miles per hour the maximum impact is found to be 33 percent, the curve formula give a percentage of 45.5 , the bridge and loading being the same as given on the bridge diagram. This percentage, according to the variation in the impact for a difference of velocity of from 40 to 50 miles per hour is evidently sufficient for a speed of 58 miles per hour.

Investigating the relation of the actual impacts and those obtained from the curve $I = 60/l$, it is found that with two exceptions the formula gives results as great or greater than the actual impact. It is probable that the two exceptions are due to instrumental errors. For a span of 300 feet and a velocity of from 25 to 30 miles per hour the impact is found to be only 1 percent lower than for a span of 210 feet and a velocity of from 30 to 40 miles per hour and with the same loading. As the percent impact should decrease more rapidly, as shown by the diagram, with the increased span length and decreased velocity it is safe to say there were errors in the test and that the actual impact was not greater than the theoretical.

Few tests have ever shown a percentage greater than 95 even for short spans, and since the velocity of the train at the time the 130 percent was obtained was perhaps greater than that which it is safe to run trains this result may be discarded.

Then one exception to the curve is made and that is as noted above.

IV. CONCLUSIONS.

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While the formulae I = 60/l fullfils the required conditions for impact when the bridge as a whole is considered it does not give values acceptable in the design of the different members.

The curves in Plate V and VI are those taken from extensometer and deflectometer tests with the instruments located upon the bridge as shown upon the respective sketches, D indicating deflectometer and those not marked by a D the extensometers. Both structures were highway bridges, that of Plate V being a 55-foot four-panel pony truss, while that of Plate VI a 70-foot five-panel thru truss. The loading was a horse and buggy traveling in the first instance with a velocity of 3 1/2 miles per hour, in the second with a velocity of 17 miles per hour.

The straight line drawn in black on each diagram is the no live load line, the curve line drawn in red is the static live load line. All readings upon each bridge were taken at the same time thus giving the percentage of impact for the members of one panel under the same conditions.

PLATE V

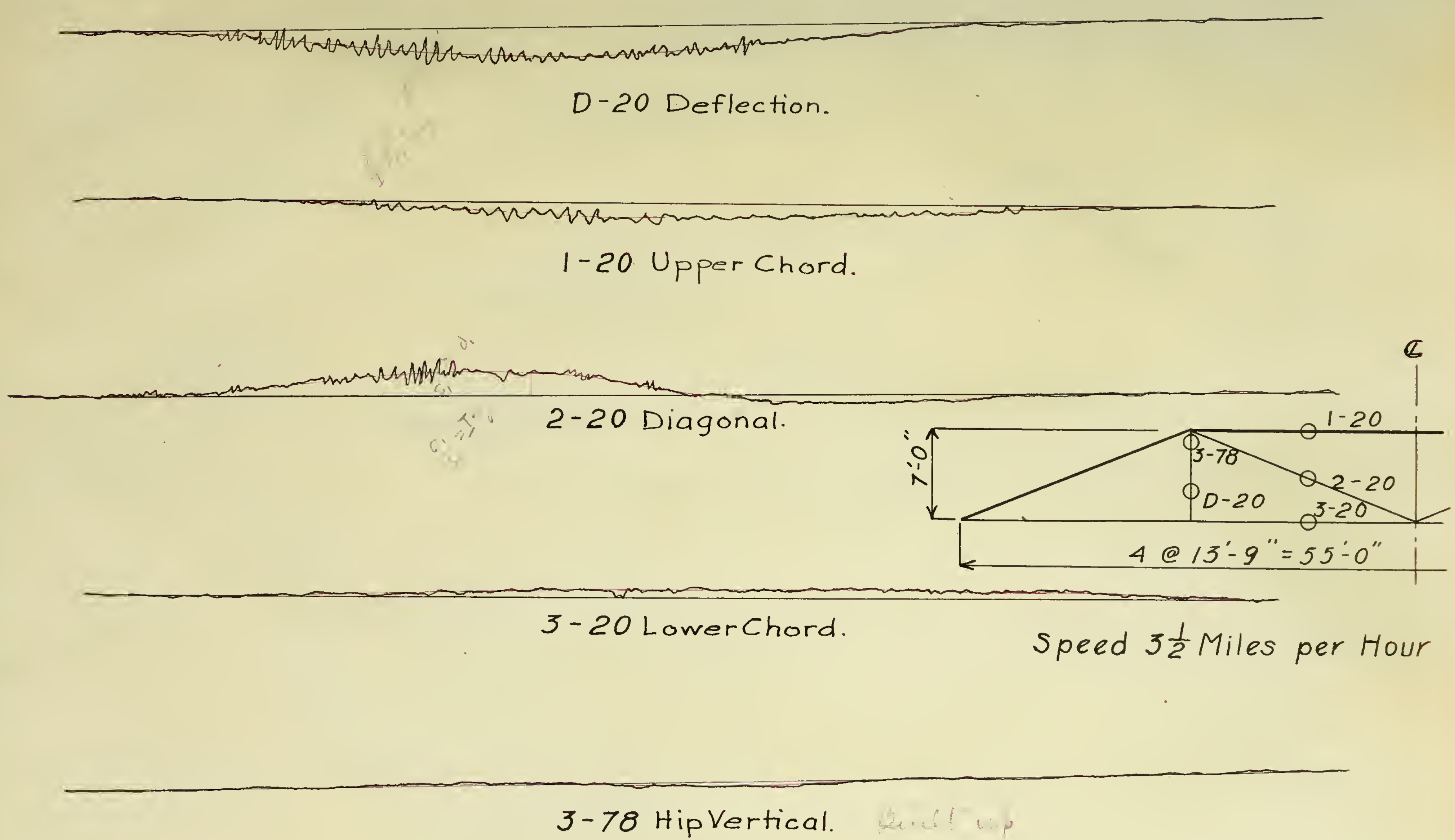
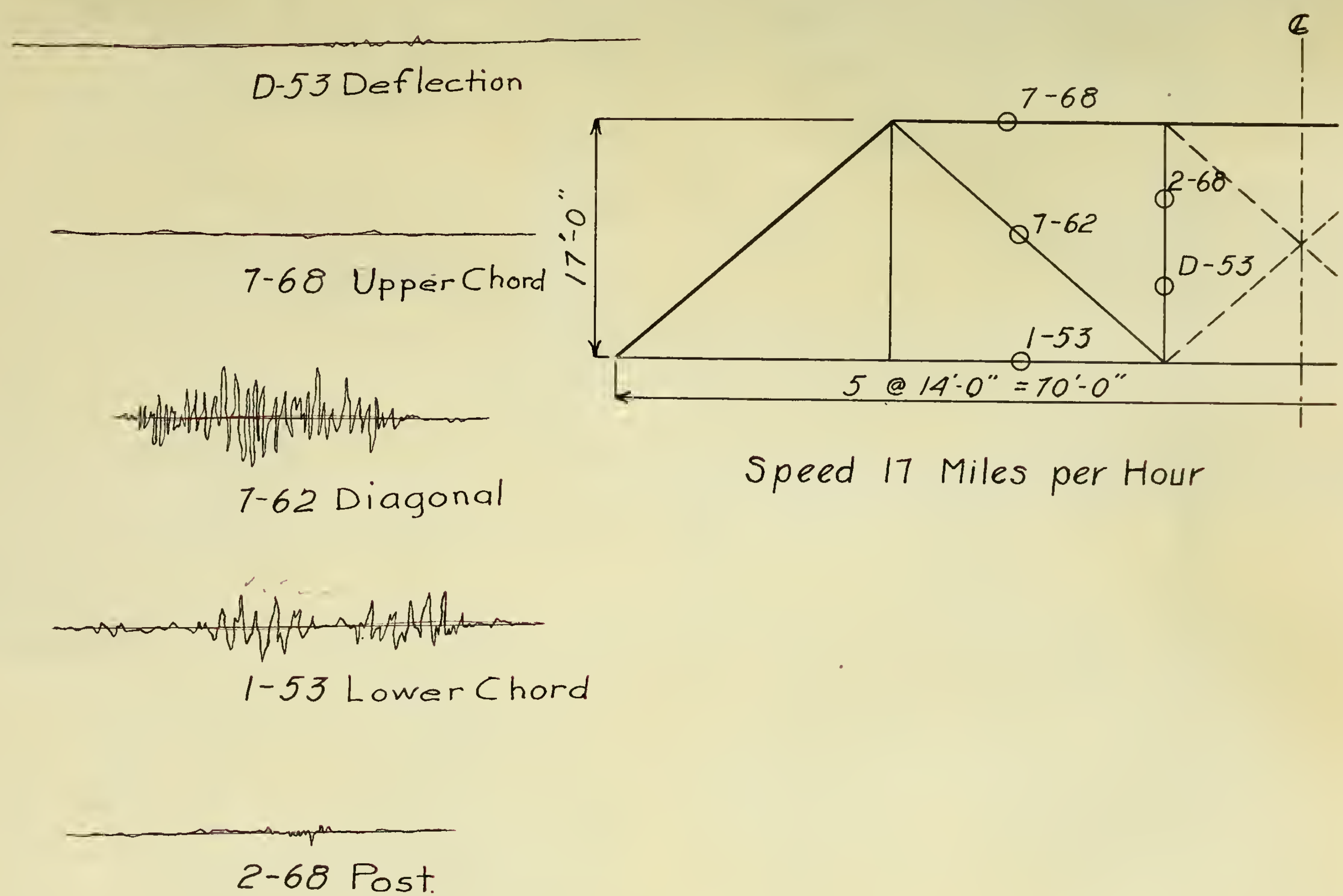

D-20 Deflection.
1-20 Upper Chord.
2-20 Diagonal.
3-20 Lower Chord.
3-78 Hip Vertical.
1-20
2-20
3-20
3-78
D-20
7'-0"
4 @ 13'-9" = 55'-0"
Speed 3½ Miles per Hour

PLATE VI



D-53 Deflection
7-68 Upper Chord
7-62 Diagonal
1-53 Lower Chord
2-68 Post.
7-68
2-68
7-62
D-53
1-53
17'-0"
5 @ 14'-0" = 70'-0"
Speed 17 Miles per Hour

Table IV.

Results of Impact Tests

Pony Truss Bridge.				Thru Truss Bridge.			
Member	Speed M.P.H.	Percentage Impact		Member.	Speed M.P.H.	Percentage Impact	
		Observed.	$I = \frac{60}{1}$			Observed.	$I = \frac{60}{1}$
Deflectometer.	$3\frac{1}{2}$	46	109	Deflectometer.	17	50	86
Hip Vert.	$3\frac{1}{2}$	20	109	First Inter.	17	117	86
Top Chord	$3\frac{1}{2}$	43	109	Lower Chord.	17	20	86
Lower Chord.	$3\frac{1}{2}$	100	109	Diagonal.	17	85	86
Diagonal.	$3\frac{1}{2}$	57	109		17	112	86

Comparing these results with those of the formula $I = 60/1$ it is seen that the maximum impact in the pony truss is less than that determined by the formula . However in the case of the thru bridge the percentage impact for the first intermediate is 1.4 and for diagonal 1.3 as great as that determined by the formula. It is seen at once that this formula is not adaptable to use in the design of members.

Further comparison may be made between the corresponding members of the two bridges. By close inspection it is seen that the percentage of deflection is nearly the same for the two bridges but the percentage of impact varies a great deal. The impact in the diagonal of the thru bridge is 2.1 times as great as that in the diagonal of the pony bridge,

of the first intermediate of the thru bridge the percentage of impact was 6 times as great as that in the hip vertical of the pony truss. With the chord members the variation was reversed. In the case of the top chord in the thru truss the percentage was found to be about 20, while in the top chord of the pony truss the percentage was 43. For the lower chord there was a difference of impact, that of the pony truss being the greater. This indicates that the deeper the truss the small the stress in the chords due to impact, but the greater the impact stress the diagonals and vertical posts.